

DISPLACEMENT-BASED SEISMIC DESIGN OF LIMITED DUCTILE RECTANGULAR RC WALLS: FROM THE DESIGN ENGINEERS PERSPECTIVE

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ABSTRACT

This paper presents an overview of how designers could undertake a displacement-based seismic assessment of rectangular reinforced concrete (RC) walls in accordance with Australian codes and standards. RC detailing in Australia generally results in a limited ductile RC structure and as such will be the focus of this paper. The various seismic performance objectives set out by Australian codes and standards are summarised herein. The paper will outline how a non-linear static pushover analysis could be performed in accordance with the Australian concrete structures standard to determine the force-displacement response curve of a rectangular wall. Mean stress-strain curves representing the actual performance of reinforcement and concrete are presented. The method presented for determining the force-displacement response curve is validated against large scale laboratory test results with very good correlation shown. The paper concludes with a parametric study of rectangular walls looking at how the overstrength and displacement ductility vary with changes in the axial load ratio, effective height, concrete strength and vertical reinforcement ratio.

KEYWORDS

Displacement-based design, reinforced concrete walls, RC walls.

INTRODUCTION

In the 90's there was extensive examination of the then current seismic design philosophy. Limitations and faults with force-based seismic design procedures were identified, and researchers and designers began considering and exploring new alternatives. Priestley (1993) authored a paper titled *Myths and Fallacies in Earthquake Engineering – Conflicts Between Design Reality* which identified and drew widespread attention to many of the problematic aspects of force-based seismic design. In the late 90's research into displacement-based seismic design began to start gaining serious momentum. *Direct Displacement-Based Seismic Design of Concrete Buildings*, written by Priestley and Kowalsky (2000), outlined the direct displacement-based design (DDBD) procedure for buildings and represented the first milestone in a gradual and not yet complete shift towards displacement-based seismic design. That paper was a precursor for the well-known text, *Displacement-Based Seismic Design of Structures* (Priestley, Calvi and Kowalsky 2007), which is internationally considered the authority on the subject matter.

While there has been much work and research in developing displacement-based seismic design procedures, currently earthquake loading standards are still largely written around force-based seismic design procedures. The most recent version of the Australian earthquake loading code, AS 1170.4 (Standards Australia 2007) included a clause allowing designers to use displacement-based assessment procedures in Australia in the form of a non-linear static pushover analysis. AS 1170.4 does not however provide any guidance to the designer wishing to use this approach. The commentary to AS 1170.4 (Wilson and Lam 2007) provides designers with further explanation, in that it says this method of analysis can be used to either (a) determine the overstrength and displacement ductility values of the structure to be used in a force-based analysis or (b) assess the performance of the structure using the 'capacity spectrum method'. The first option allows for structure specific overstrength and ductility values to be assessed as opposed to using the default values given by the code. This overcomes one of the major criticisms of force-based analysis in that it is assumed all structures of a similar basic structural form and level of detailing possess the same level of overstrength and ductility (Priestley 2013; Priestley et al. 2007). The second option allows for the structure to be assessed using the capacity spectrum method, outlined in detail by Wilson and Lam (2007), where the earthquake performance of a structure is assessed by overlaying its capacity curve on an acceleration-displacement response spectrum (ADRS).

AIMS AND OBJECTIVES

This paper will present a procedure design engineers could use to assess the earthquake compliance of reinforcement concrete (RC) buildings in Australia using a displacement-based design approach; while complying with all relevant Australian codes and standards. The performance levels of structures required at different limit state loading scenarios in the relevant codes of practice are discussed, followed by an examination of the information required to perform non-linear analyses of RC walls in accordance with the Australian concrete structures standard, AS 3600 (Standards Australia 2009). A procedure will then be presented for undertaking a non-linear static pushover analysis without the need of a specialised finite element (FE) package. The focus of this paper will be limited ductile RC walls as they form the primary elements of the lateral load resisting system in the majority of low, mid and high-rise structures built in Australia.

CODE PERFORMANCE LEVELS

The Building Code of Australia (BCA) (Australian Building Codes Board 2014) outlines the earthquake loading requirements for buildings constructed in Australia. The BCA stipulates a return period of earthquake actions which buildings must be designed for based on their importance level. Buildings are assigned an importance level from 1 to 4 depending on their intended use: importance level 2 is for normal buildings; importance level 3 is for buildings that may contain large numbers of people; and importance level 4 is for buildings that are required for post-disaster recovery or may contain hazardous materials. In addition to the requirements of the BCA, AS 1170.4 requires importance level 4 buildings to undergo a special study to demonstrate they remain serviceable after an importance level 2 equivalent event. The intent of the latter, while it is not explicitly written in AS 1170.4, is that the building is operational following the event, with limited non-structural damage permitted; unlike a serviceability wind event where no damage, either structural or non-structural, should occur. Figure 1 outlines the earthquake performance criteria required by the BCA and AS 1170.4. Importance level 1 buildings are not discussed here because very few buildings or structures fall into this category.

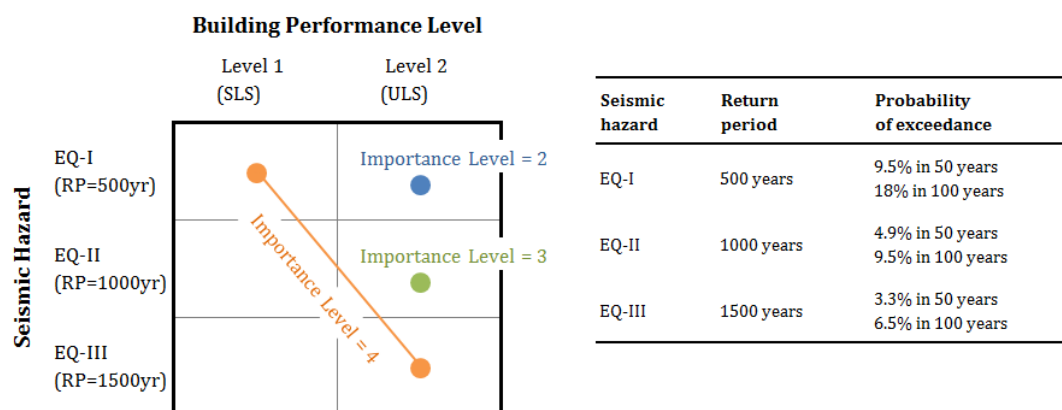


Figure 1 Australian earthquake performance criteria – design level earthquakes.

There are no documents that explicitly outline inter-storey drift limits for buildings in Australia at different performance levels. Although some standards do contain clauses stating an inter-storey drift limit for a specific level of performance. AS 1170.4 requires an inter-storey drift limit of 1.5 per cent for the ultimate limit state (ULS) performance of buildings. The commentary to the Australian concrete structures standard, AS 3600 Suppl (Standards Australia 2014) says the inter-storey drift limit of concrete buildings under serviceability limit state (SLS) conditions should be 0.2 per cent. The latter recommendation however is only appropriate for wind loading where the SLS event is typically a 1 in 25 year return period event. For an earthquake SLS event (i.e. for importance level 4 buildings) an appropriate inter-storey drift limit would be 0.4 to 0.5 per cent. The inter-storey drift limits are summarised in Table 1.

AS 3600 specifies material strain limits for RC structures. It provides strain limits for ultimate limit state performance but does not provide any guidance for serviceability limit state performance. It stipulates that the ultimate compressive strain limit of concrete should be 0.003 when using the ‘rectangular stress block’ assumption. For advanced methods of analysis taking into account material stress-strain relationships AS 3600’s only requirement for compressive strain limits is that “the strain in the compressive reinforcement does not exceed 0.003”. No tensile strain limits are specified and hence tensile reinforcement strain limits can be taken as the ultimate strain of the reinforcement (ϵ_{su}). The strain limits are summarised in Table 2.

Table 1 Australian earthquake performance criteria – inter-storey drift limits.

Standard	Serviceability limit state (SLS)	Ultimate limit state (ULS)
AS 1170.4	–	1.5 %
AS 3600 Suppl	0.2 %	–
Recommendation by the authors for the seismic performance of importance level 4 buildings	0.4 to 0.5 %	1.5 %

Table 2 Australian earthquake performance criteria – AS 3600 material strain limits.

Material limit	Serviceability limit state (SLS)	Ultimate limit state (ULS)
Concrete	–	–
Compressive reinforcement	–	0.003
Tensile reinforcement	–	ϵ_{su}

While AS 3600 does not place a strain limit on the tensile reinforcement, the reinforcement in walls subject to cyclic lateral load is generally unable to develop its ultimate tensile strain without low cycle fatigue or local buckling failures occurring prior. Sullivan, Priestley and Calvi (2012) recommend Eq. 1 for the tensile limit of reinforcement under ULS equivalent loading. Priestley et al. (2007) stipulates that for vertical reinforcement to achieve the tensile strains expressed by Eq. 1 without the bars buckling they need to be restrained by ligatures at a maximum spacing of $s = (3 + 6(f_{su}/f_{sy}))d_b$, where d_b is the bar diameter of the vertical reinforcement.

$$\epsilon_{s,uls} = \max[0.6\epsilon_{su}; 0.05] \quad (1)$$

RC walls constructed in Australia typically have non-ductile detailing without confinement reinforcement (i.e. ligatures). Typical detailing consists of a mat of equal sized and spaced vertical and horizontal bars each face with extra ‘U’ bars at the ends of walls and around openings. The authors are currently undertaking experimental testing to better understand the maximum strain the vertical reinforcement in limited ductile RC walls can undergo prior to bar buckling occurring. The current testing suggests reinforcement strains can exceed 3 per cent prior to the onset of this behaviour. For the moment a tensile strain limit of 0.03 is being recommended.

MEAN STRESS-STRAIN CURVES

AS 3600 requires all non-linear analysis methods to use “mean values of all relevant material properties” and AS 3600 Suppl further states that for “non-linear and other refined methods of analysis, actual stress-strain curves, using mean rather than characteristic values, should be used”. AS 3600 provides guidance to what the mean in-situ strength of concrete is but no guidance is provided to what a suitable mean stress-strain response curve of reinforcement is. Menegon et al. (2015) have recently undertaken a study to determine the mean properties of reinforcement complying with AS/NZS 4671 (Standards Australia and Standards New Zealand 2001). Bilinear stress-strain curves were recommended for the expected (i.e. mean) response of D500L, D500N and D500E reinforcement. These curves were based off the tensile test results of over 6,000 rebar samples obtained from an independent materials testing laboratory that is contracted by industry suppliers to test and ensure code compliance of their rebar. The values for constructing the bilinear stress-strain curves are summarised in Table 3. The bilinear stress-strain curve for D500N reinforcement – the typical and most commonly used grade of reinforcement in Australia – that is recommended for use in non-linear analyses is shown in Figure 2.

Table 3 Mean properties of reinforcement (Menegon et al. 2015).

Grade	Yield stress (f_{sy})	Ultimate stress (f_{su})	Ultimate strain (ϵ_{su})
D500L	585 MPa	620 MPa	3.3 %
D500N	550 MPa	660 MPa	9.5 %
D500E	530 MPa	660 MPa	13 %

Menegon et al. (2015) also provide a discussion as to what the long term in-situ strength of concrete is. With reference to Neville (1996), they recommended there is no long term strength gain of in-situ concrete. This is in contrast to continuously moist cylinder samples which continue to strengthen somewhat indefinitely. The mean in-situ strength of concrete (f_{cmi}) is taken to be 90 per cent of the mean strength of concrete (f_{cm}) as per the recommendations in AS 3600 and AS 3600 Suppl1 (Eq. 2). AS 3600 Suppl1 recommends a modified version of the Thorenfeldt, Tomaszewicz and Jensen (1987) model for the unconfined stress-strain response of normal and high strength concretes. It has been modified to suit Australian concretes and is shown in Eq. 3. The modified Thorenfeldt et al. (1987) model is shown in Figure 2 for standard grades of concrete used in RC walls.

$$f_{cm} = (1.2875 - 0.001875f'_c)f'_c \quad (2)$$

$$f_c = f_{cmi} \left[\frac{n(\varepsilon_c/\varepsilon_{co})}{n-1+(\varepsilon_c/\varepsilon_{co})^{nk}} \right] \quad (3)$$

Where: ε_c is the concrete strain; ε_{co} is the concrete strain corresponding to the maximum compressive stress and is taken as $4.11f_{cmi}^{0.75}/E_c$; $n = E_c/(E_c - E_{sec})$; E_c is the modulus of elasticity specified by AS 3600, i.e. 30100, 32800, 34800 and 37400 for N32, N40, N50 and S65 concrete grades respectively; E_{sec} is the secant modulus of elasticity and is equal to f_{cmi}/ε_{co} ; When $\varepsilon_c \leq \varepsilon_{co}$, $k = 1$ and when $\varepsilon_c > \varepsilon_{co}$, $k = 0.67 + \varepsilon_{co}/62 \geq 1.0$.

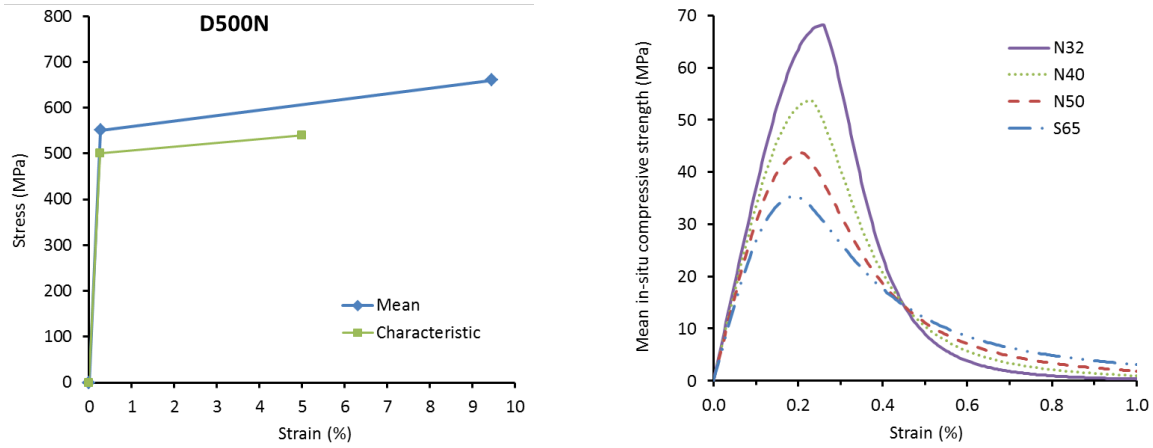


Figure 2 LEFT: Mean and characteristic stress-strain curves of D500N reinforcement. RIGHT: Mean in-situ unconfined stress-strain curves for standard grades of concrete used in RC walls.

NON-LINEAR STATIC PUSHOVER ANALYSIS

The capacity curve (i.e. force-displacement response) of a limited ductile RC wall can be determined by firstly undertaking a moment-curvature analysis of the section. The force-displacement response is then calculated from the moment-curvature response based on the height of the wall and the concept of a plastic hinge, where over a plastic hinge length (L_p) a uniform curvature distribution equal to the maximum curvature of the wall is assumed (refer Figure 3). The process explained herein is for slender RC walls that are dominated by flexural actions, i.e. the overall height to length ratio is greater than two to three.

Lam, Wilson and Lumantarna (2011) presented a method for determining the moment-curvature response of a section using Microsoft Excel spreadsheets. This approach enables designers to determine the complex non-linear behaviour of cracked RC members without the need for any specialised software packages (e.g. Response-2000 (Bentz and Collins 2001)). The material stress-strain relationships presented in the previous section are recommended for undertaking moment-curvature analyses on RC structures built in Australia. The non-linear moment-curvature response of the section can be simplified to an approximate bilinear response curve. This is done by projecting a line from the origin through the point of the response curve corresponding to the notional yield curvature (ϕ'_y) and up to the point corresponding to the yield curvature (ϕ_y) of the section. The corresponding moment value at this location is the nominal moment capacity (M_n). A straight line is then projected from this point to the point corresponding to the ultimate curvature (ϕ_u) of the section (refer Figure 4).

With reference to AS 3600, where applicable, and Priestley et al. (2007) the following criteria is being recommended. The notional yield curvature (ϕ'_y) is taken as the point corresponding to the first yield of the extreme tensile reinforcement (i.e. $\varepsilon_s = \varepsilon_{sy}$) or the maximum compressive stress being reached in the extreme compressive fibre (i.e. $\varepsilon_c = \varepsilon_{co}$), whichever occurs first. The yield curvature (ϕ_y) is taken as the point

corresponding to the strain in the extreme tensile reinforcement reaching 0.015 or the maximum compressive stress of the concrete being reached in the extreme compressive fibre (i.e. $\varepsilon_c = \varepsilon_{co}$), whichever occurs first. The concrete strain limit for the yield curvature and notional yield curvature are taken as the same value because confinement ligatures are not used in RC walls in Australia (typically). The ultimate curvature (ϕ_u) is taken as the point corresponding to the tensile reinforcement reaching 0.03 (as discussed in a previous section) or the strain in the extreme compressive vertical reinforcement reaching 0.003, whichever occurs first. The slope of the elastic branch of the bilinear relationship is equal to $E_c I_{eff}$ and as such the effective second moment of area (I_{eff}) of the wall can be calculated using Eq. 4.

$$I_{eff} = \frac{M_n}{E_c \phi_y} \quad \text{OR} \quad I_{eff} = \frac{M_y}{E_c \phi'_y} \quad (4)$$

The bilinear moment-curvature response can be converted to force-displacement using the process and equations proposed by Priestley et al. (2007) shown by Eqs 5 through 8. That is, the yield displacement (Δ_y) is calculated using the maximum yield curvature at the base and assuming a linear curvature distribution up the height of the wall. The ultimate displacement (Δ_u) is calculated using the plastic hinge concept as is illustrated in Figure 3. The plastic hinge length incorporates a strain penetration length (L_{sp}) of the starter bars into the supporting element below the wall. Priestley et al. (2007) provide a detailed explanation to why integrating the actual curvature distribution of an RC wall (i.e. using first principles of structural mechanics) does not produce displacement values that agree well with experimental tests.

$$F = \frac{M}{H_e}; \quad F_y = \frac{M_y}{H_e}; \quad F_n = \frac{M_n}{H_e}; \quad F_{max} = \frac{M_{max}}{H_e} \quad (5)$$

$$\Delta_y = \frac{\phi_y (H_e + L_{sp})^2}{3} \quad (6)$$

$$\Delta_u = \Delta_y + (\phi_u - \phi_y) L_p \left[H_e - \left(\frac{L_p}{2} - L_{sp} \right) \right] \quad (7)$$

$$L_p = k H_e + 0.1 L_w + L_{sp} \quad (8)$$

Where: H_e is the effective height of the wall and can be taken to equal 70 per cent of the overall height of the wall; L_{sp} is the strain penetration and can be taken to equal $0.022 f_{sy} d_b$; L_w is the wall length; and $k = 0.2(f_{su}/f_{sy} - 1) \leq 0.8$. (Priestley et al. 2007).

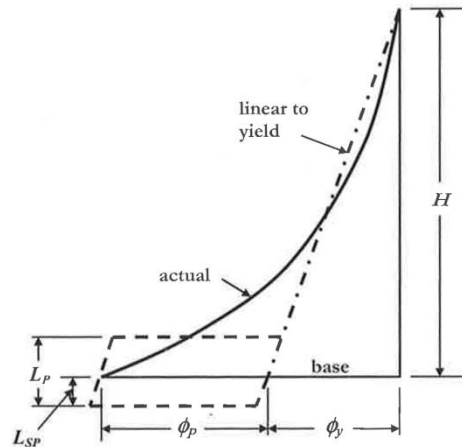


Figure 3 Idealisation of curvature distribution (Priestley et al. 2007).

After constructing a force-displacement response curve, overstrength and ductility factors can be determined. The overstrength factor (Ω) is taken to be the maximum force capacity of the structure (F_{max}) divided by the ultimate force capacity calculated in accordance with AS 3600 ($\phi F_u = \phi M_u / H_e$), i.e. $\Omega = F_{max} / \phi F_u$. Note in this instance ϕ is a capacity reduction factor which is used in ULS design to Australian material standards. AS 1170.4 accounts for the overstrength by the use of the structural performance factor (S_p), where $S_p = 1/\Omega$ (Wilson and Lam 2007). The displacement ductility is determined using Eqs 9 and 10.

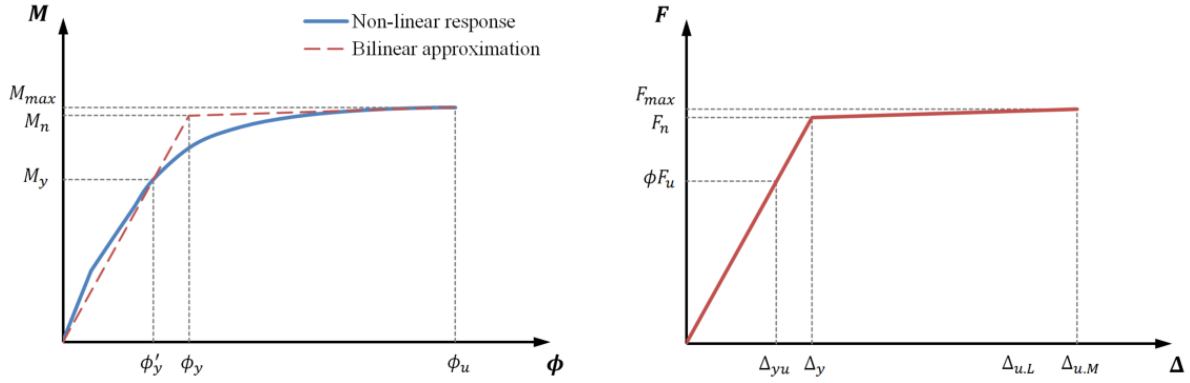


Figure 4 LEFT: Non-linear moment-curvature response with bilinear approximation.
RIGHT: Bilinear force-displacement response

$$\mu_{\Delta} = \frac{\Delta_u}{\Omega \Delta_{yu}} \quad (9)$$

$$\Delta_{yu} = (\phi F_u) \left(\frac{\Delta_y}{F_n} \right) \quad (10)$$

The effective stiffness of the wall (k_{eff}) can be calculated by dividing the nominal force capacity by the yield displacement, i.e. $k_{eff} = F_n / \Delta_y$.

The process outlined so far yields a bilinear force-displacement response curve for an RC wall. Priestley et al. (2007) also offers another approach to determine a ‘refined’ force-displacement response curve; expressed by Eqs 11 to 13.

$$\text{Cracking: } \Delta_{cr} = \frac{\phi_{cr} H_e^2}{3} \quad \text{and} \quad F_{cr} = \frac{M_{cr}}{H_e} \quad (11)$$

$$\text{First yield: } \Delta'_y = \frac{\phi'_y (H_e + L_{sp})^2}{3} \quad \text{and} \quad F_y = \frac{M_y}{H_e} \quad (12)$$

$$\text{After yield: } \Delta_{cr} = \Delta'_y \left(\frac{M}{M_y} \right) + \left[\phi - \phi'_y \left(\frac{M}{M_y} \right) \right] L_p \left[H_e - \left(\frac{L_p}{2} - L_{sp} \right) \right] \quad \text{and} \quad F = \frac{M}{H} \quad (13)$$

To assess the accuracy of the outlined approach with the material stress-strain curves suggested (i.e. bilinear model for reinforcement and AS 3600 Supp1 for concrete), a validation against an experimental laboratory test of a large scale RC wall was performed. There has been many experimental laboratory studies undertaken worldwide looking at the in-plane lateral loading behaviour of RC walls. Unfortunately the majority of these studies have either (a) studied the behaviour of ductile RC walls (i.e. walls with confinement reinforcement in the end compressive regions of the walls) or (b) been performed on squat RC walls where shear behaviour dominates the response. Very few studies have looked at the flexure response of slender limited ductile RC walls. Dazio, Beyer and Bachmann (2009) recently undertook an experimental study that included one test wall that almost exactly matched the detailing and construction practices in Australia. The test wall was 2000 mm long, 150 mm thick and had an aspect ratio of approximately 2.3. The reinforcement had similar stress-strain properties to Australian reinforcements and the wall was tested with an axial load ratio of about 4 per cent. The cross section of the test wall is shown in Figure 5.

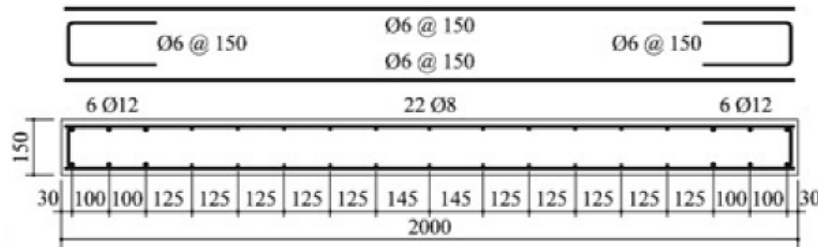


Figure 5 Dazio et al. (2009) test specimen WSH4.

The wall was initially analysed using the ‘refined method’ proposed by Priestley et al. (2007), as outlined previously, using the AS 3600 Suppl1 unconfined concrete stress-strain model. The theoretical curve almost exactly matches the enveloped cyclic behaviour of the wall, about up to the point where the compressive strain in the reinforcement reaches 0.003. After which the theoretical model slightly underestimates the performance, as shown in Figure 6. The analysis was repeated using the classical Mander, Priestley and Park (1988) confined concrete stress-strain model. It was calculated that the ‘U’ bars at each of the wall would provide a small amount of lateral confinement of 0.5 MPa – calculated in accordance with equations proposed by Mander et al. (1988) for lateral confinement – which results in an increase in compressive strength of approximately 6 per cent. It can be seen in Figure 6 that the predicted response up to the compressive strain limit being reached is almost identical between the models, however following this point the Mander et al. (1988) model more accurately predicts the response of the test wall.

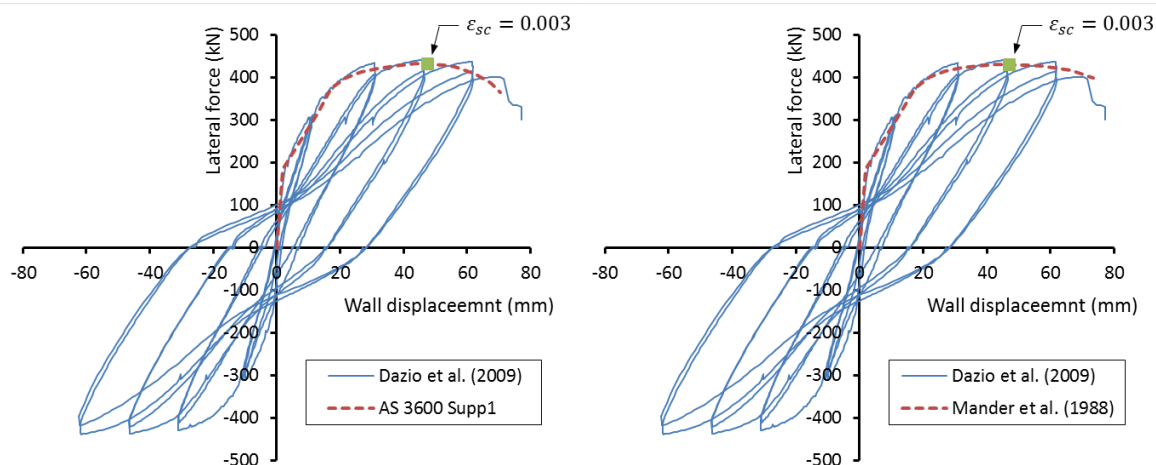


Figure 6 Comparison of the experimental and theoretical force-displacement response for wall specimen WSH4 using (a) the AS 3600 Suppl1 unconfined model and (b) the Mander et al. (1988) confined model.

The AS 3600 Suppl1 unconfined concrete stress-strain model is being recommended for general use in modelling limited ductile RC walls for a variety of reasons, including:

1. Ultimate response to AS 3600 limits the compressive reinforcement strain to 0.003 and at this level of performance there is no observable difference between the results from the two stress-strain models.
2. While it is extremely common for RC walls in Australia to be detailed with ‘U’ bars in the end regions of walls, it is not a codified requirement and hence some walls do not have them. This would effectively eliminate the added performance on the post peak branch of the response curve.
3. The AS 3600 Suppl1 model yields only slightly more conservative values beyond the compression reinforcement strain limit of 0.003 and so the added complexities of using the confined Mander et al. (1988) model are generally not justified.

SENSITIVITY STUDY

Part of the requirements of undertaking a non-linear analysis to AS 3600 is the designer must allow for variability of material properties. The commentary, AS 3600 Suppl1 states that the results of a non-linear analysis should be checked by undertaking a sensitivity study where the designer runs a series of analyses “in which the key input variables are altered systematically”. This activity was performed for a typical rectangular wall that was 3000 mm long and 250 mm thick with a concrete grade of N40, axial load ratio of 5 per cent and a vertical reinforcement ratio of 1.1 per cent.

The parameters which were varied included: the yield and ultimate stress of the reinforcement; the ratio of the ultimate to yield stress of the reinforcement; the ultimate strain of the reinforcement; the axial load on the wall; and the in-situ compressive strength of the concrete. An initial analysis was performed using the mean values for all the parameters, as discussed previously. Additional analyses were performed where each of the parameters was changed from the mean value to the lower and upper characteristic values. Except for the axial load on the wall, this was varied by plus or minus 20 per cent. No more than 2 parameters were changed at once. The results of the study can be seen in Figure 7. The ultimate curvature varied from 16 per cent smaller to 20 per cent higher than the mean response reference value and the corresponding moment varied from 7 per cent smaller to 9 per cent higher than the mean response reference value.

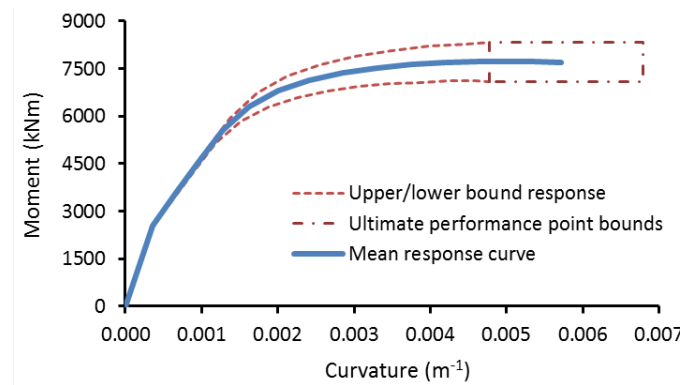


Figure 7 Sensitivity study: moment-curvature results.

It can be seen that for the rectangular wall used in the study changing the individual material properties of either the reinforcement or concrete, or varying the axial load, does not greatly affect the overall performance of the wall. It is important to note that using the criteria proposed in this paper generally results in the ultimate performance of limited ductile rectangular RC walls being governed by the compressive strain limit being developed in the compression reinforcement. As such the ultimate tensile strain of the reinforcement used in the analysis does not affect the performance of the wall; changing the mean ultimate strain (9.5 per cent) to the lower or upper characteristic values of 5.75 and 13.18 per cent respectively had no observable effect on the ultimate curvature. This is in contrast to sections with a large compressive flange (i.e. 'T' sections or box-shaped lift cores) where the tensile strain in the tension reinforcement is generally the limiting criteria. For these sections varying the ultimate strain of the reinforcement can have very dramatic effects on the ultimate displacement and displacement ductility of the section, as shown by Menegon et al. (2015).

PARAMETRIC STUDY

A parametric study using the non-linear static pushover analysis method presented earlier was performed to better understand the overstrength and displacement ductility of limited ductile rectangular RC walls. Initially two walls were considered; one being 3000 mm long and 250 mm thick and the other being 5000 mm long and 250 mm thick. Both walls had an axial load ratio of 5 per cent, an effective height of 25 m, a vertical reinforcement ratio of 1.4 per cent and a concrete grade of N40. After determining the overstrength and displacement ductility of these two baseline walls, each of the four aforementioned parameters were varied and the new overstrength and displacement ductility values were calculated. The results are shown in Figure 8.

It was observed that the displacement ductility generally decreases with an increase in the axial load ratio, effective height or vertical reinforcement ratio of the wall. Increasing the concrete strength seems to slightly increase the ductility; however this increase in ductility has a corresponding slight decrease in overstrength. The only parameter which appears to significantly affect the overstrength is the axial load ratio of the wall and is largely due to the capacity reduction factor used in ULS design to AS 3600, which equals 0.8 when a wall is in pure bending and decreases to 0.6 when the axial load equals the balanced load of the wall.

For limited ductile RC walls AS 1170.4 recommends an overstrength value of 1.3 and a displacement ductility value of 2. The combination of the two parameters (i.e. $\Omega\mu_{\Delta}$) forms the reduction factor used in force-based seismic analysis and in this case equals 2.6. The overstrength was generally 5 to 20 per cent higher than the value of 1.3 proposed by AS 1170.4. In contrast, the displacement ductility was calculated to be between approximately 1.2 and 1.6, which is less than the value of 2 proposed by AS 1170.4. This results in the reduction factor (i.e. $\Omega\mu_{\Delta}$) being constantly less than 2.6. It should be noted here that failure in this instance is typically the AS 3600 compressive reinforcement strain limit being reached, not axial load failure of the wall (i.e. collapse).

The walls with an effective height of 25 m (e.g. a wall in an 8 to 12 storey building) had an ultimate displacement that was always greater than 250 mm (refer Figure 9). The peak displacement demand (PDD) of the AS 1170.4 response spectrum on a 'very soft soil' site (i.e. class E_e) for a typical ULS earthquake (i.e. RP = 500 years) in Melbourne or Sydney is 90 mm. The latest amendment of AS 1170.4 (in press) requires a multiplication factor of 1.5 to the displacement demand when performing displacement-based assessments, resulting in a PDD of 135 mm. This value can then be multiplied by 1.6 to account for any torsional response of asymmetric floor plans (Lumantarna, Lam and Wilson 2013). This results in a 'worst case' displacement demand for an ULS event of 216 mm. Indicating automatic code compliance for all of these walls, despite the displacement ductility being less than the value of 2 given in AS 1170.4.

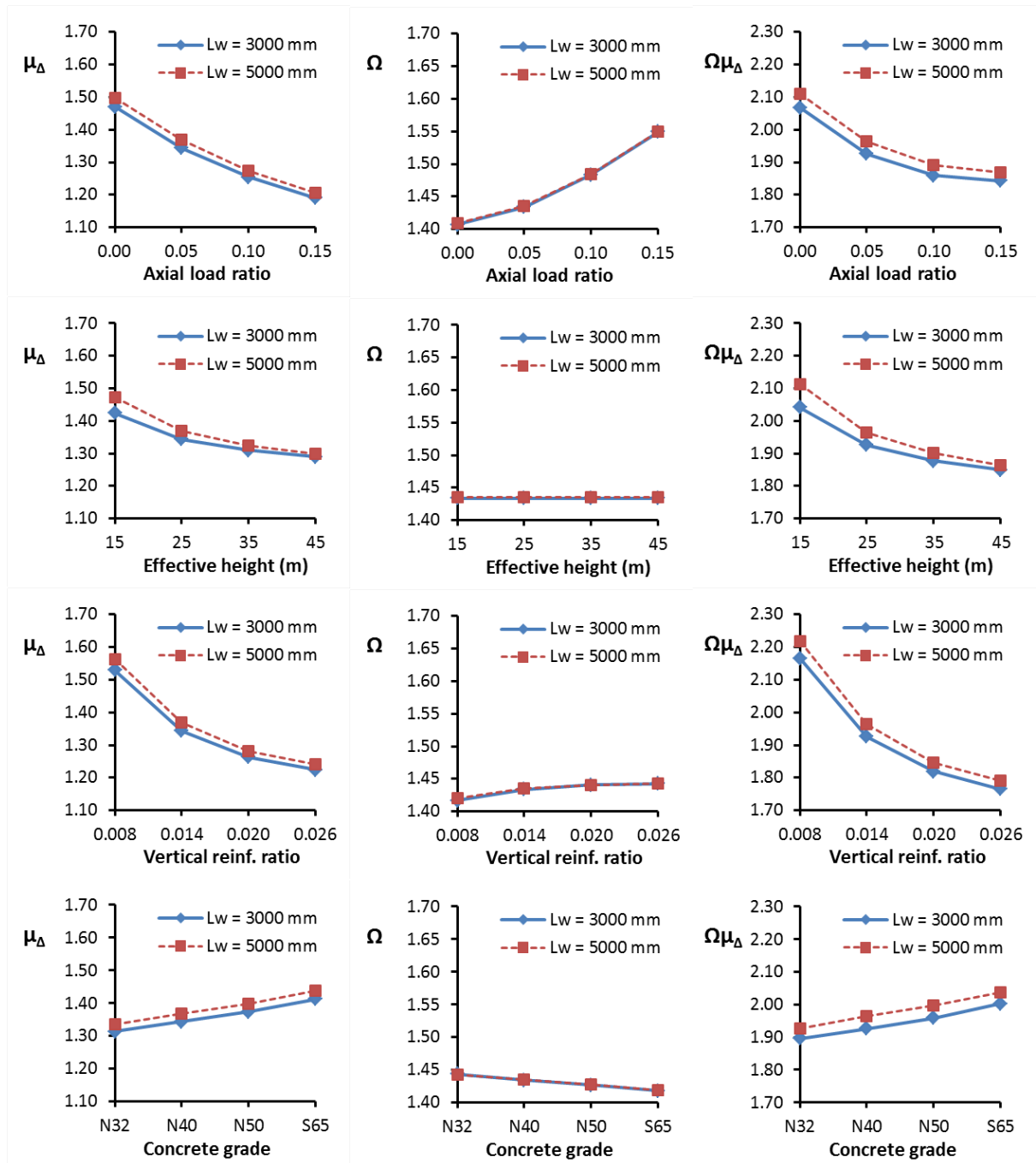


Figure 8 Parametric study results. LEFT: Displacement ductility. MIDDLE: Overstrength. RIGHT: Overstrength * displacement ductility.

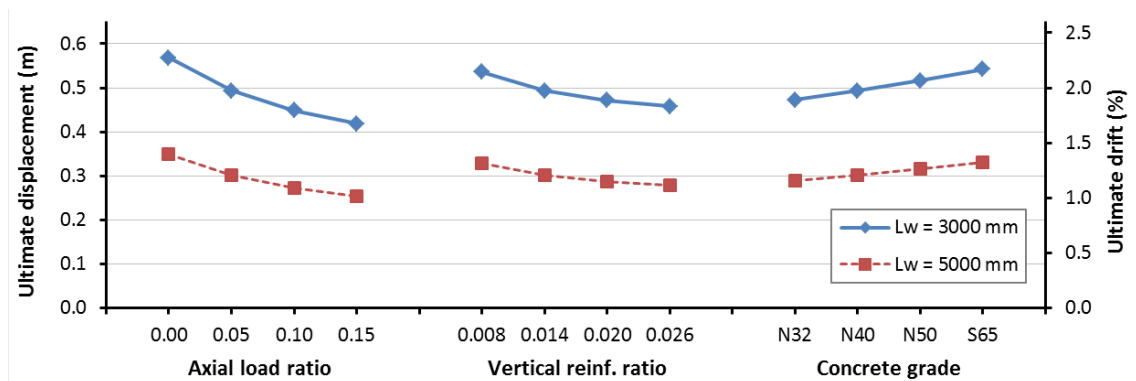


Figure 9 Ultimate displacement and ultimate drift for rectangular RC walls with an effective height of 25 m.

CONCLUSION

This paper has presented a design procedure practicing structural engineers could potentially use to undertake a displacement-based seismic assessment of rectangular limited ductile RC walls, in accordance with AS 1170.4 and AS 3600. A non-linear static pushover analysis and mean stress-strain curves of concrete and reinforcement have been presented. The analysis procedure has been validated against a large scale experimental laboratory test of a limited ductile RC wall. A preliminary sensitivity study has been presented and indicates that varying the material properties of reinforcement and concrete does not greatly affect the performance of rectangular walls. A parametric study was undertaken to better understand how the overstrength and displacement ductility of these walls vary with respect to axial load ratio, effective height, vertical reinforcement ratio and concrete strength. It was shown the overstrength was always greater than the value of 1.3 given in AS 1170.4, whereas the displacement ductility was much less than the codes value of 2.

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